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Settlement monitoring of shallow foundations in collapsible tropical soil

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Article

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Abstract

The presence of collapsible soils poses a significant risk for foundation pathologies and is prevalent in various parts of the world, including tropical regions. This paper evaluates the behavior of a specific tropical collapsible soil through laboratory tests and field monitoring of foundations over the course of one year. Laboratory results indicate that the soil has a high collapse potential, particularly at stress levels exceeding 200 kPa, where it is considered critically problematic. Field monitoring revealed that external columns experienced settlement increases of up to 2.7 mm following rainfall events. In contrast, internal columns or those protected by concrete surfaces exhibited less seasonal variation in settlement, with changes of around 1 mm. These observations highlight that long-term exposure to moisture can result in undesirable settlement and underscore the need for moisture considerations in foundation design for collapsible soils. Structures that are initially loaded and subsequently exposed to moisture are likely to experience accelerated future settlement, potentially leading to structural pathologies. Therefore, consideration of moisture effects is critical in foundation design to mitigate long-term settlement issues.

1. Introduction

In geotechnical engineering, estimating settlements in shallow foundations has been the subject of extensive studies since the mid-1940s. The literature on this topic presents several methods for predicting settlement, which can be categorized into three main types: i) Empirical Methods: These methods correlate settlement estimates directly with field test data. Notable examples include the studies by Terzaghi & Peck (1948), which established foundational empirical approaches; ii) Strain Influence and Elasticity-Based Methods: These approaches use factors that influence strain (Schmertmann, 1970) or are grounded in the theory of elasticity (Bowles, 1987; Burland & Burbidge, 1985; Mayne & Poulos, 1999); iii) Instrumented Load Tests and Numerical Simulations: These modern methods provide detailed settlement curves based on load tests and numerical simulations. Examples include the work of Schneider-Muntau & Bathaeian (2018), Wu et al. (2020) and Stewart et al. (2023). Methods (i) and (iii) are commonly used for estimating settlements of isolated footings, while method (ii) is more appropriate for foundations with large contacts such as raft foundations.

Pathologies related to settlement of foundations are commonly reported around the world. Various factors contribute to these issues, including design and construction errors (Haydl & Nikiel, 2000; Tan et al., 2020), inadequate soil investigation (Fenton et al., 2005; Vilar & Rodrigues, 2015), the presence of underground cavities (Liu et al., 2019; Mahdavi, 2020), discontinuities of geological formations (Elkateb et al., 2003; Shi & Wang, 2022) and the presence of collapsible soils. Among these, collapsible soils are a particularly significant factor in foundation pathologies (Du et al., 2014; Kalantari, 2013; Li et al., 2016; Opukumo et al., 2022; El-Shafee et al., 2024; Bandeira et al., 2024).

Collapsible soils were first identified and studied by Jennings & Knight (1975). These soils experience a rapid decrease in volume when subjected to stress and subsequent increases in water content (Vilar & Rodrigues, 2015; Hashim et al., 2023; AlNaddaf et al., 2024). Collapsible behavior is typical in unsaturated soils with low specific weight and can occur in various soil types, including residual, colluvial, aeolian, alluvial and poorly compacted soils (Pereira & Fredlund, 2000; Houston et al., 2001; Rollins & Kim, 2010; Vilar & Rodrigues, 2011; Brink & van Rooy, 2015; AlNaddaf et al., 2024). In arid and semi-arid

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regions the formation of collapsible soils is exacerbated by unsaturated conditions and specific formation processes (Lawton et al., 1992; Houston et al., 2001; Hashim et al., 2023; Bandeira et al., 2024).

Many studies have utilized laboratory techniques to qualitatively assess the potential for soil collapse, based on factors such as particle size distribution, Atterberg limits, and specific weight. Quantitative evaluations often involve double oedometer tests or oedometer tests with saturation (Jennings & Knight, 1975; Phien-wej et al., 1992; Phanikumar et al., 2016; Wang et al., 2020). These methods determine collapse potential by measuring deformation due to saturation at a given stress, or by comparing strain differences between natural and saturated samples in double oedometer tests (Jennings & Knight, 1975).

While laboratory tests offer greater accuracy and control over variables, they have limitations, such as sampling challenges and reduced representativeness of field conditions. Laboratory conditions often fail to replicate the complexities of the field environment, including voids and moisture variations. Additionally, simulating the stress state of soils and the conditions of water pore pressure or suction in the laboratory is challenging. This disconnect highlights the ongoing gap between scientific geotechnical research and practical engineering where scientific studies rely on controlled laboratory tests, while practical engineering depends on field performance data and associated empirical studies (Peck, 1979; Bennett et al., 2011; Fonseca e Pineda, 2017).

Field studies addressing soil collapsibility have employed plate load tests with varying water content or with modifications to water content during loading (Houston et al., 2001; Phien-wej et al., 1992; Vianna et al., 2007; AlNaddaf et al., 2024). Comparisons between field measurements and laboratory tests often reveal significant differences, with laboratory tests typically showing greater settlements than field studies (El-Ehwany & Houston, 1990; Ferreira & Fucale, 2014; Hashim et al., 2023).

Despite advancements in improving the representativeness of collapse studies, there are still limited records of monitoring shallow foundations in collapsible soils. Monitoring vertical and horizontal displacements is crucial for evaluating the performance of full-scale geotechnical works, particularly in larger construction projects (Bennett et al., 2011; Smethurst et al., 2017; Stewart et al., 2023). Such field monitoring enables comprehensive analysis of soil behavior and can enhance predictions regarding settlements in future construction.

In this context, the aim of this study is to monitor the settlement behavior of shallow foundations under collapsible tropical soil conditions. In addition, laboratory tests are conducted to evaluate soil collapsibility, and the results are compared with field measurements.

2. Materials and methods

2.1 Location and description of the study area

The study area is located in Barreiras, a city in the western part of the state of Bahia, Brazil. The region has a tropical savannah climate, ranging from humid to dry sub-humid, with well-defined seasons, drought from May to September (representing 6% of the annual rainfall), with an average annual rainfall of 900 to 1500 mm (INMET, 2025).

Universidade Federal do Oeste da Bahia (UFOB)'s campus is located southwest of the city's urban perimeter, covering an area of 34 ha on the right bank of the Rio de Ondas (Figure 1).

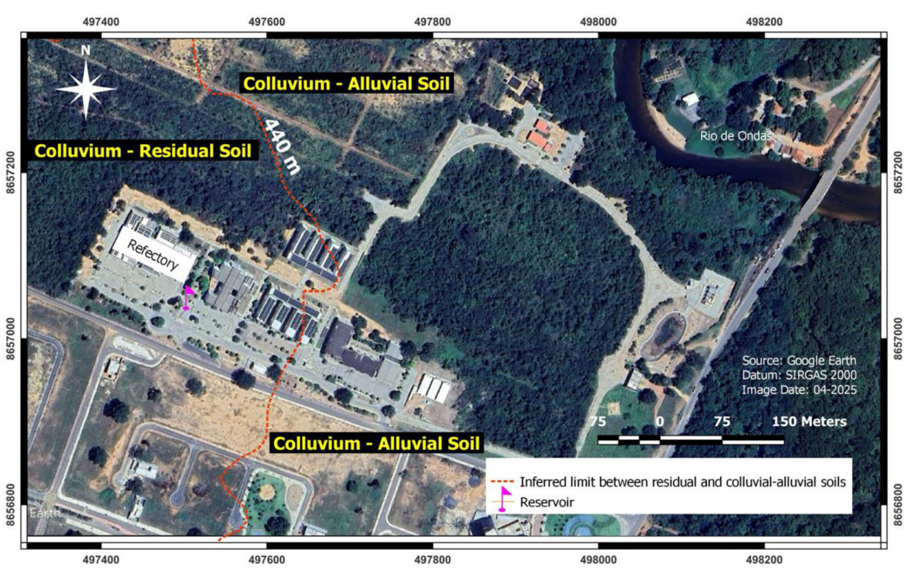

Figure 1. Location of the study area and inferred boundary between colluvium-residual soil and colluvial-alluvial soils.

The campus is located between gently undulating terrain and flat areas formed by alluvial soils deposited by the Rio de Ondas and the Rio Grande. The soils in the region include colluvial soils composed of fluvio-eolic sandstones from the Posse and Serra das Araras formations of the Urucuia Group and the Chapadão do Urucuia formation, as well as siltstones and marine clay from the Serra da Mamona and Riachão das Neves formations of the Bambui Group. On the campus's relatively flat topographical features, the colluvial coverage is fine-grained, exposing residual silty-clay soils originating from the weathering of metapelitic rocks (metasiltites and metargilites) in the Serra da Mamona Formation (Barros et al., 2020; Dantas et al., 2022).

The study focuses on two buildings: a refectory and an elevated reservoir (Figure 1). The refectory foundation consists of 84 isolated footings, with dimensions ranging from 1.2 x 1.2 .m to 2.4 x 2.4 m, each seated at least 2 m deep in relation to the level of the natural terrain. In the reservoir, there are four associated footings measuring 3.6 x 3.6 m and embedded to a depth of 3.0 m.

Soil samples, both disturbed and undisturbed, were collected from a depth of 3 m for laboratory tests. These samples represent the same soil underlying the foundations of the structures.

2.2 Sample characterization and laboratory methodologies

The characterization of the soil samples was conducted in accordance with established standards: D 854 (ASTM, 2014), D 4318 (ASTM, 2017a) and D 7928 (ASTM, 2017b). The tests yielded the following properties: a liquid limit (w_L) of 31.0%, a plastic limit (w_P) of 26.7%, and a plasticity index (PI) of 4.3%. The specific gravity (G_s) was found to be 2.73. Based on these parameters, the soil is classified as low compressibility silt (ML) according to the Unified Soil Classification System (ASTM, 2017c).

Particle size distribution is illustrated in Figure 2 and was determined following standard D 7928 (ASTM, 2017b) using wet sieving methods. This procedure was carried out both with and without dispersant to assess the aggregation content of the tropical soil, as indicated by previous studies (Miguel & Vilar, 2009; Miguel & Bonder, 2012; Otálvaro et al., 2015; Ng et al., 2020). The test with dispersant revealed a higher clay content (42%); followed by silt (18%), fine sand (15%), medium sand (15%) and gravel (10%). Conversely, the test without dispersant showed a lower clay fraction and a higher silt fraction, indicating that clay particles tend to aggregate with silt fractions.

Oedometer tests were conducted on undisturbed samples under two conditions: saturated and at natural water content. These tests followed standard D 2435 (ASTM, 2020) and involved vertical stresses of 6, 12, 25, 50, 100, 200, 400 and 800 kPa during the loading stage, with subsequent unloading at 200, 50 and 12 kPa.

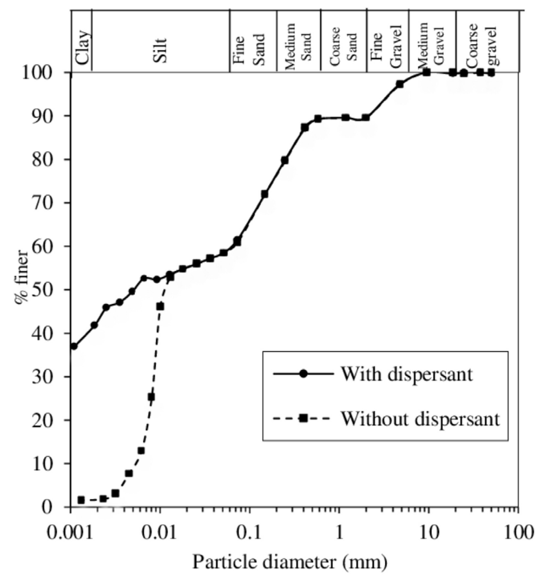

Figure 2. Particle size distribution with and without dispersant.

2.3 Methodology for settlement monitoring

Settlement monitoring was conducted over the course of a year on 14 isolated footings, 10 in the refectory and 4 in the reservoir, as shown in Figure 3a. Figure 3b illustrates a cross-section with the support dimensions of an isolated footing embedded in the ground (Figure 3c). Footings were selected based on the highest loads (200 kPa) and included both internal and external footings. The photos taken during construction (Figure 3c, d) and after construction (Figure 3e) show the refectory and the elevated reservoir structure.

Once the footings were completed, parabolts (Figure 4a) were anchored in the columns and fixed at a height of 0.2 m from the floor, centred on the column's face, with approximately 5 cm exposed (Figure 4b). Above each parabolt, a measuring system was set up (Figure 4c).

Settlement measurements were carried out using calibrated geometric levelling with an accuracy of 0.7 mm/km, which is appropriate for this study. Eight benchmarks (BMs) were installed for this purpose. The BMs were made of concrete in a quadrangular shape, measuring 0.3 x 0.3 m, with 0.1 m buried underground. A 12.5 mm^2 CA-50 steel rod, 1 m long, was embedded in each BM to prevent displacement. Throughout the monitoring period, no elevation changes were detected between the BMs. The BMs were strategically placed around the building to facilitate measurement of the monitored columns and remained unobstructed throughout the study (Figure 3a). These BMs formed vertices of a closed polygon with the settlement parabolts as reference points. Angular and linear errors in the polygon were corrected, and the X, Y and Z coordinates of the BMs and parabolts were determined. Levelling and re-levelling procedures were conducted based on the reference dimension (Z) to ensure consistency throughout the monitoring period.

Figure 3. (a) Location of reference points and isolated footings; (b) General support elevation; (c) Isolated footing; (d) Refectory during construction; (e) Buildings after construction.

Settlement measurements were taken by comparing the levels of the parabolts with those of the BMs. Readings were obtained horizontally using a level positioned vertically on each parabolt (Figure 4c). An automatic Wild Heerbrugg NA2 geometric level, calibrated to an accuracy of 0.7 mm/km, was used for these measurements. Weekly readings were recorded over the one-year period, covering intervals before, during and after load applications.

Load distributions were estimated proportionally according to the work progress, as referenced by Gusmão Filho (2002) and Naghibi & Fenton (2022). Pluviometric data were sourced from the National Institute of Meteorology (INMET, 2025) automatic station Barreiras-A402, 2,650 m from the study area.

3. Results and discussion

3.1 Compressibility and Collapsibility of the soil

The compressibility curves for the samples under natural and saturated conditions are shown in Figure 5a, and the potential for collapse is illustrated in Figure 5b. The parameters obtained from these tests are summarized in Table 1.

In the saturated condition, the compressibility index (C_c) was slightly higher compared to the natural soil, indicating a minor change in the virgin compressive section. However, there was a notable decrease in the the pre-consolidation stress due to the increase in water content. The pre-consolidation stress was calculated using both the Casagrande ($\sigma_{pc\ C.G.}$) and Pacheco Silva ($\sigma_{pc\ P.S.}$) methods (ASTM, 2020), with similar results obtained from both methods.

The potential for collapse, as depicted in Figure 5b, was calculated using the approach proposed by Jennings & Knight (1975). This involved adjusting the compressibility curve of the natural sample by translating it vertically to match the field stress and voids ratio. The severity of collapse is assessed based on the applied field stress: at 100 kPa, the soil is considered problematic, while at stresses of 200, 400 and 800 kPa the soil is classified as having a severe collapse problem. Qualitative analysis using physical indices following the methods of Gibbs & Bara (1967) also supports the conclusion of soil collapse.

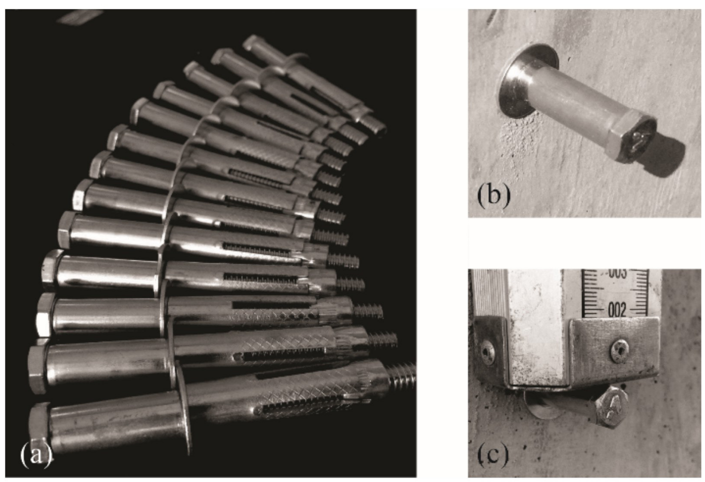

Figure 4. (a) Parabolts; (b) Anchored parabolt; (c) Wild Heerbrugg NA2 level positioned on the parabolt.

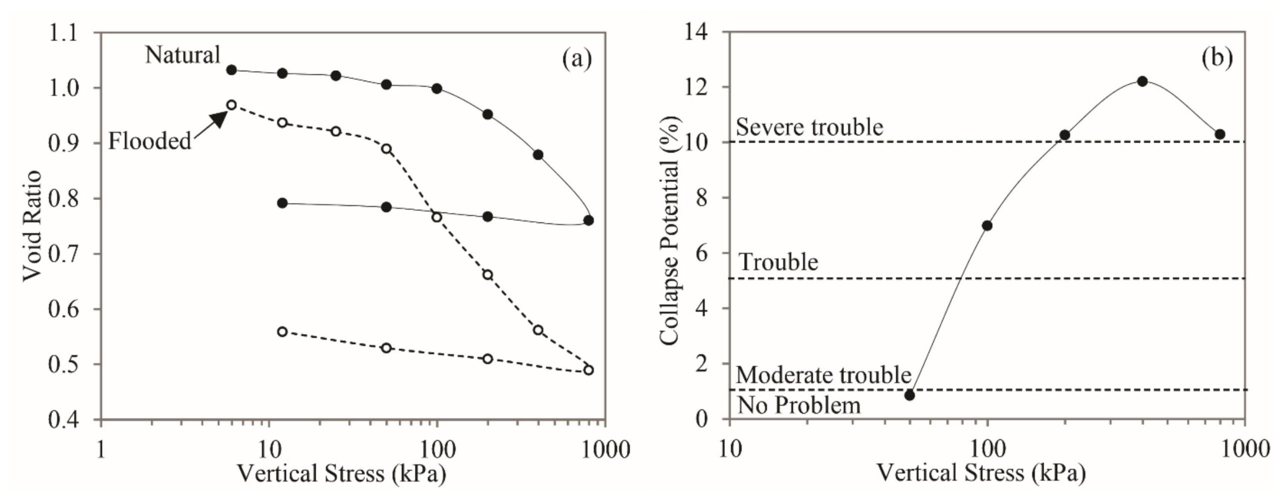

Figure 5. Results of tests for evaluation of compressibility and collapse: (a) double oedometer test; (b) potential for collapse and severity of the problem.

Table 1. Parameters obtained from the oedometer tests.

Sample	C_c (kPa^{-1})	C_r (kPa^{-1})	$\sigma_{pc\ C.\ G}$ (kPa)	$\sigma_{pc\ P.\ S.}$ (kPa)
Natural	0.319	0.020	108	105
Saturated	0.339	0.040	50	45

$\sigma_{pc\ C.G.}$, pre-consolidation stress calculated according to the Casagrande method. $\sigma_{pc\ P.S.}$, pre-consolidation stress calculated according to Pacheco Silva method (ASTM, 2020)

In Brazil, collapsible soils are prevalent in several states (Vianna et al., 2007; Vilar & Rodrigues, 2011, 2015; Bandeira et al., 2024) and often have metastable pore structure with both macro and micro pores, which are formed due to weathering in well-drained regions (Miguel & Bonder, 2012; Otálvaro et al., 2015; Cordão Neto et al., 2018).

Collapsible soils can originate from various sources, including residual, colluvial, alluvial and aeolian deposits, such as silty loess soils (Rollins & Kim, 2010; Brink & van Rooy, 2015; Vilar & Rodrigues, 2015; Wang et al., 2020). These soils are classified under different categories in the Unified Soil Classification System (USCS). The soil in this study is identified as low compressibility silt (ML) and is classified as colluvial-residual soil. However, collapse in these soils is often influenced more by their initial unsaturated condition and microstructural factors than their origin. Factors such as porous structure, weak particle bonds and aggregations contribute significantly to collapsibility (Li et al., 2016), and can be present in various soil types.

Oedometer tests conducted on the soil samples in both natural and saturated conditions allowed for the analysis of vertical deformations and displacements as shown in Figure 6. The stress-strain curves reveal distinct behaviors under the two conditions. In the natural condition, the curve is non-linear at lower stresses, but approaches linearity as the stress reaches 100k Pa, indicating that displacements become proportional to the applied normal stresses. Conversely for the saturated condition, the stress-strain curve shows greater deformations at the same stress levels compared to the natural soil. The nonlinearity is more pronounced up to 100k Pa, with a noticeable shift to linear behavior at stresses above 200 kPa.

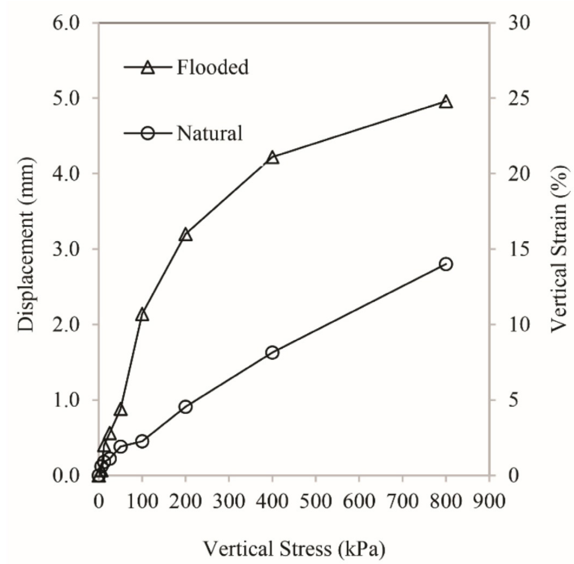

Figure 6. Stress-strain curves for the saturated and natural soil samples.

3.2 Settlement monitoring

Figure 7a and 7b show the measured field settlements. Monitoring started on day zero for the refectory columns and on day 71 for the elevated reservoir, due to the differing construction start times. The data reveals that most settlements began to stabilize after 71 days (see Figure 7a), with columns P26, P29, P35 and P36 showing settlement of 1.0 mm. The relatively quick stabilization suggests that the settlements are primarily influenced by the granular colluvial soil, which is composed of sand and gravel and constitutes 45% of the particle size distribution.

In contrast, columns P81 and P85 experienced the largest settlements, exceeding 2.3 mm (Figure 7b). This area corresponds with the lowest elevation on the site, where the level difference between P7 and P85 is 1.57 m. The largest settlements at P81 and P85 are attributed to the additional vertical loads imposed by an embankment added to the footing. Despite their lower elevation, there is no accumulation of water at these locations due to effective surface drainage. Overall, the observed settlements are not expected to cause any structural issues.

By day 273, after the full load had been applied, the settlement magnitudes across most columns had stabilized and showed minimal variation, even among those columns with higher loads, which were installed later.

Figure 8 presents the iso-settlement curves at the end of the monitoring period, generated using Surfer software and the kriging method. Columns P26, P29, P35 and P36, exhibited stable settlements of 1.0 mm, occurring between days 22 and 29. The P1 and P7 columns showed minimum variability in their settlements over time, with P1 reaching maximum settlement of 1.3 mm and stabilizing from day 71, and P7 reaching a maximum settlement of 1.5 mm with stability from day 57. P61 and P70 recorded maximum settlements of 1.2 and 1.5 mm, respectively.

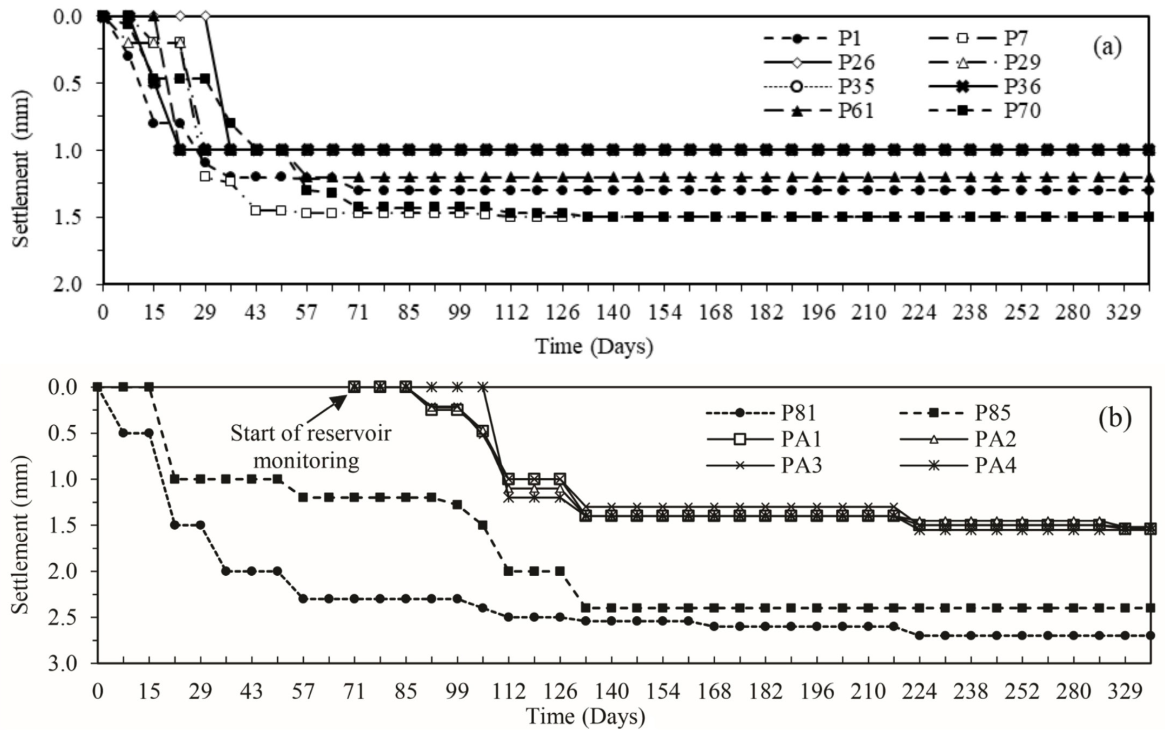

Figure 7. Settlement measured in the field: (a) P1, P7, P26, P29, P35, P36, P61, P70; (b) P81, P85, PA1, PA2, PA3, PA4.

3.3 Relationship between loading, rainfall, and settlements

Figure 9 and Figure 10 illustrate the relationship between settlement, rainfall, and loading on the columns. The columns are depicted symmetrically with respect to the design and subjected to uniform loading over time.

Figure 9 focuses on columns P1, P7, P26 and P29, which are comparable due to their extended period without additional loads in the initial weeks of construction. Notably, rainfall events between the 3rd and the 9th week played a significant role in influencing settlements. For instance, P1 experienced a 60% load (≈120 kPa) in the first week, resulting in a settlement of 0.8 mm by the 3rd week, followed by stabilization. In contrast, P7, with the same loading, exhibited a smaller settlement of 0.2 mm. Rainfall in the subsequent weeks accelerated settlements, increasing by 0.5 mm in P1 and 1.1 mm in P7. Load increases for these columns only occurred in the 18th week.

Despite variations in loading and moisture content affecting columns P1, P7, P26 and P29 (Figure 9 and Figure 10), no significant settlements were observed beyond the 64th week. Notably, between the 64th and 100th weeks, sidewalks and pavements with interlocking blocks were installed on the left side of the layout (P1 and P7) extending to columns P26 and P29 (Figure 3a). As a result, rainfall after day 105 did not significantly alter moisture levels or reduce soil bearing capacity under these columns. Settlements recorded in dryer conditions were similar to those observed under wetter conditions. This finding is corroborated by Figure 6, which demonstrates that the laboratory displacement for the saturated sample at 50 kPa, 0.88 mm is nearly identical to the displacement of the natural sample at 200 kPa, 0.91 mm.

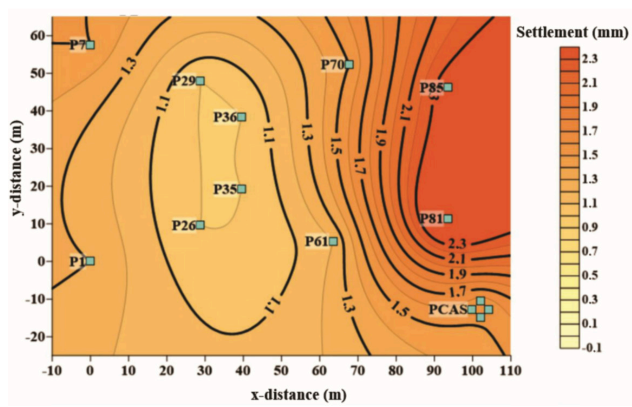

Figure 8. Iso-settlement curves.

Although the external columns bear less load, soil-structure interaction affects the settlements of the foundations. These interactions result in load redistribution among the structural elements, with exceptions for load transfer from more heavily loaded footings to those with the lighter loads. However, this behavior was not observed consistently, making the impact of precipitation on columns P1 and P7 more apparent. According to Farouk & Farouk (2016) the rigidity of the footing and the superstructure contributes to lower magnitudes of the differential settlements, as seen in columns P26 and P29.

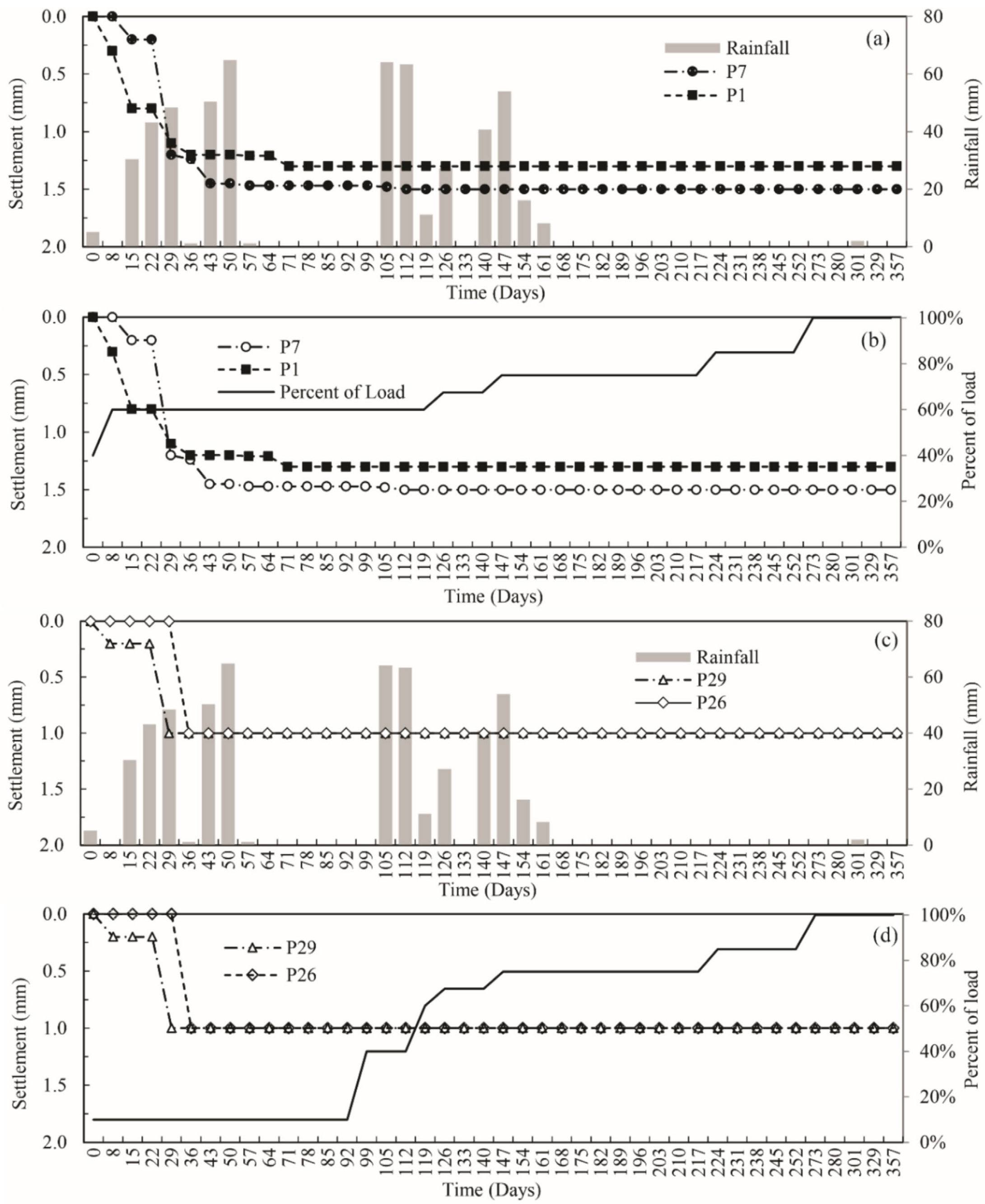

Figure 9. Settlement and relation to: (a) rain in P7 and P1; (b) loading in P7 and P1; (c) rain in P29 and P26; (d) loading in P29 and P26.

Figure 10 presents the settlement behavior of the PA2 and PA4 reservoir columns which are positioned at the extremities of the reservoir and are affected simultaneously by loading and rainfall. Due to their proximity and structural rigidity, the settlement behaviors of these columns are quite similar. The settlement of these columns can be divided into four distinct stages:

1. 1st Stage: On day 92, the settlement measured 0.3 mm. At this point, the first floor of the building, including columns and beams, had been completed, and there was no recorded rainfall for that week. This suggests that the observed settlement was solely due to the applied loading.
2. 2nd Stage: Between days 112 and 126, the settlement increased to 1.1 mm. During this period, the second floor was completed, and the cumulative rainfall amounted to 101.8 mm. It is important to note that rainfall in the two weeks preceding this period was

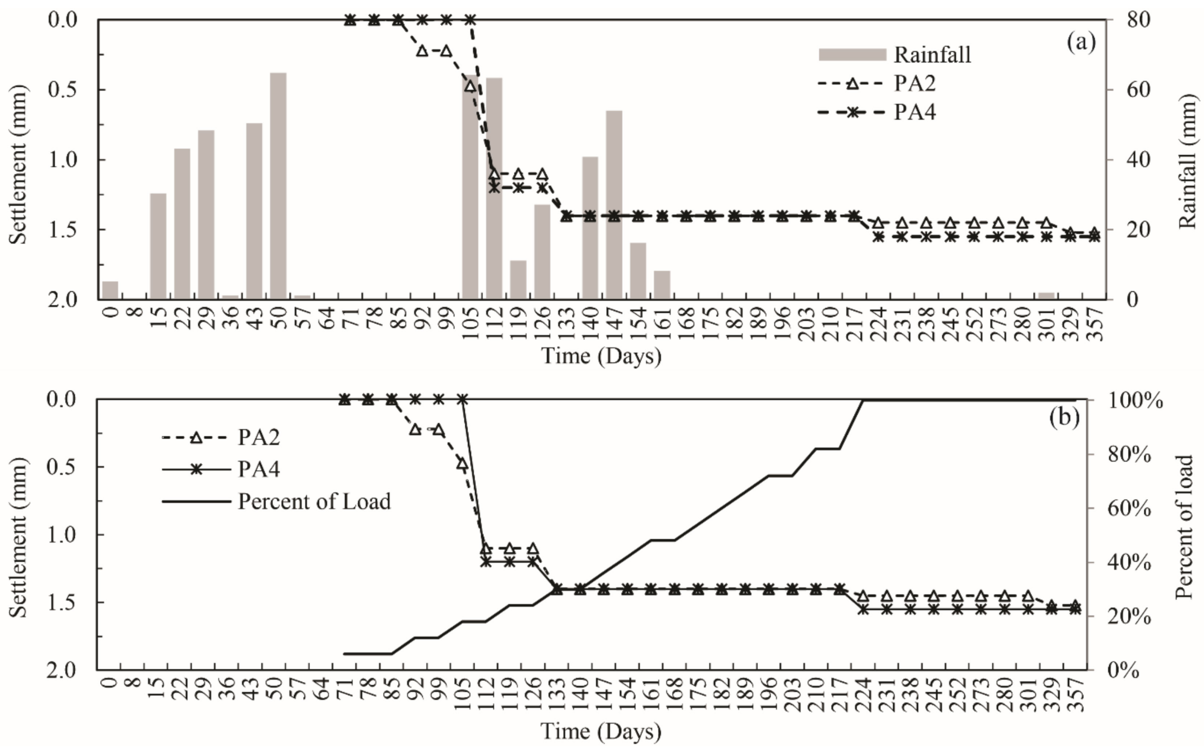

Figure 10. Settlement and relation to: (a) rain in PA2 and PA4; (b) loading in PA2 and PA4.

also significant, which likely contributed to the observed settlements.

3. 3rd Stage: From days 140 to 217, the settlement reached 1.4 mm. During this time, the top floors of the reservoir were constructed, and the rainfall accumulation was 119.2 mm between days 140 and 161. The settlement remained stable at 1.4 mm throughout this stage.
4. 4th Stage: Between days 224 and 357, the settlement varied from 1.5 mm to 1.6 mm between days 224 and 357. At the beginning of this stage, the reservoir was filled to its operational level. Rainfall during this period was negligible to analysis of the settlement. Initially there was a 0.1 mm settlement due to water loading, with an additional increase of 0.1 mm observed on day 329.

Overall, initial settlements were significantly influenced by rainfall, as the early load applied was considerably lower than the final load. It is important to consider that the pre-consolidation stress of natural soil was 108 kPa, representing 55% of the total load, while the pre-consolidation stress of the saturated soil was 50 kPa (25% of the total load). The proportion of loading relative to the pre-consolidation stress accounts for 96% of the settlement observed in the PA2 column and 90% in the PA4 column.

4. Comparison between field settlements and laboratory tests

Figure 11 presents the displacements obtained from the natural and saturated oedometer tests and the settlements measured in the field. The results are presented for two scenarios: 50% of the loading (approximately 100 kPa in the laboratory as depicted in Figure 11a) and 100% of the loading (200 kPa in the laboratory as shown in Figure 11b).

When comparing the saturated condition as a reference, laboratory displacements are consistently higher than field measurements across all stress levels, except for the P81 column at 100 kPa. Conversely, when the natural condition is used as a reference, field displacements generally exceed the laboratory results. This indicates that the settlement in the field at the depth of interest (2 m) during rainy periods is lower than under the saturation condition in laboratory tests, but higher than under the natural condition observed in the oedometer test. In other words, the laboratory saturation condition can be considered overly extreme and does not accurately reflect the in-situ moisture conditions of this soil.

Notably, the internal columns, exhibit smaller field settlements compared to their laboratory counterparts. The close alignment of these field measurements with the natural oedometer test results suggests that these columns benefit from protection against external moisture due to their location within the building.

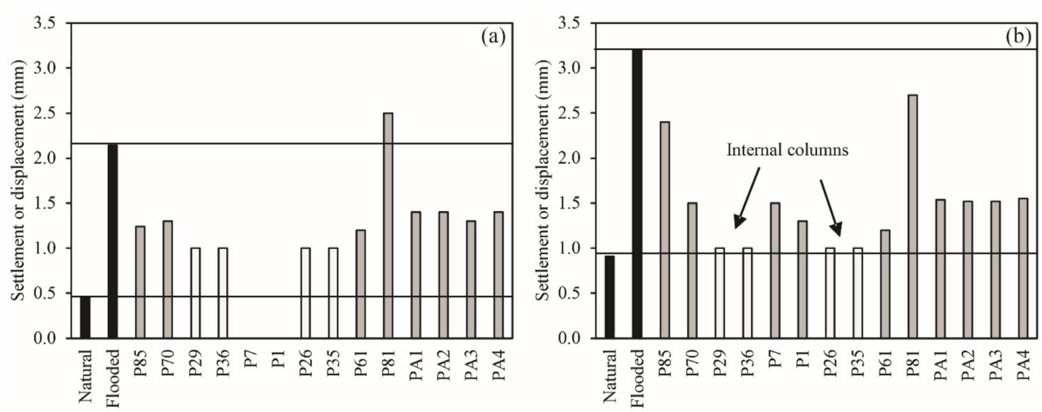

Figure 11. Laboratory displacements and field settlements: (a) 50% of the load; (b) 100% of the load.

5. Conclusions

Collapsible soils can lead to significant structural issues in buildings, such as cracks and severe failures. These soils are found in various regions around the world. This research aimed to monitor the settlement of shallow foundations in a collapsible tropical soil and compare the results with laboratory tests.

The laboratory oedometer tests revealed significant differences in soil behavior, particularly with regard to pre-consolidation stress, which decreased by 50% with increasing water content. Laboratory tests indicated that soil had a high potential for collapse, being classified as problematic under 100 kPa stress and critical under 200 kPa.

Field measurements taken over one year using geometric levelling before and after construction recorded maximum settlements ranging from 1.0 to 2.7 mm with consistent increases over time. Therefore, it should be noted that the settlements in the field were not of great magnitude and were not significantly affected by rainfall. The building's rigidity played a crucial role in stabilizing these settlements.

When comparing the final monitored settlements to the 200 kPa stress tests, field settlements fell within the range of displacements observed for both natural and saturated samples. This suggests that the laboratory saturation condition can be considered overly extreme and does not reflect the field moisture conditions for this soil.

The field tests demonstrated the impact of loading and rainfall on soil settlement. External columns that experienced long periods without load variation, showed increased settlement after rainy periods. In contrast, internal columns, protected by concrete surfaces, exhibited less seasonal variation due to moisture, as they were less affected by environmental conditions. This suggests that the soil subjected to increased moisture and loading tends to accelerate future settlements, which could mitigate potential structural pathologies. However, increased moisture after additional loading stages may lead to undesirable settlements. Thus, moisture-related changes should be considered in the design of foundations for collapsible soils.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have reviewed and approved the content of the paper and have no financial interests to declare.

Author's contributions

Paulo Vinícius do Nascimento Martins: Conceptualization, Data curation, Visualization, Material preparation and data collection, Analysis, Writing – original draft. José Luiz Ernandes Dias Filho: Visualization, Writing – review & editing, Resources. Luis Gomes Carvalho: Conceptualization, Data curation, Material preparation and data collection, Methodology, Analysis, Project administration, Resources. Mauro Alexandre Paula de Sousa: Conceptualization, Visualization, Analysis. Vinícius de Oliveira Kühn: Conceptualization, Visualization, Methodology, Analysis, Writing – review & editing, Supervision, Project administration.

Data availability

The datasets generated analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

h	Height
w_L	Liquid limit
w_P	Plastic limit
BM	Benchmark
C_c	Compressibility index
C_r	Recompression index
G_s	Specific gravity
ML	Low-compressibility silt
P	Pillar
PA	Reservoir pillar
PI	Plasticity index
UFOB	Universidade Federal do Oeste da Bahia
USCS	Unified Soil Classification System
X	x-coordinate
Y	y-coordinate
Z	z-coordinate
$\sigma_{pc\ C.G.}$	Pre-consolidation stress calculated according to the Casagrande method.
$\sigma_{pc\ P.S.}$	Pre-consolidation stress calculated according to Pacheco Silva method

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